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MODEL TESTING OF RECOVERY LOADS FOR DECOMMISSIONING SKIRTED SUBSEA STRUCTURES

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Abstract
Decommissioning of subsea structures founded on shallow foundations currently requires the complete removal of the structure, but there are challenges in predicting the associated recovery loads. This work examines the extraction behaviour of shallow, skirted, square foundations in very soft clay by means of a series of displacement-controlled small scale model tests. Skirt length, uplift rate, surface drainage condition and preload were all varied. Extraction behaviour is seen to be different depending on whether the foundation retains a soil plug which in turn seems to depend on the kinematics rather than rate effects. Prediction of recovery loads based on assuming plugged behaviour in all cases provided an upper bound without excessive conservatism. Prediction of recovery loads based on model preload was only successful under certain conditions, and thus should be considered as an estimate only.

1. Introduction

To carry vertical, lateral, moment and torsional loads efficiently on the seabed, subsea structures are often supported by skirted footings, such as Figure 1. As the North Sea and West of Shetland basins mature, a number of offshore oil and gas fields where this type of structure have been installed have become, or will shortly become, “life expired”. Under OSPAR Decision 98/3, subsea structures must be removed from the seabed and recovered to shore as part of a field decommissioning programme. However, recovery of these structures is technically challenging and carries significant risk due to uncertainties in the magnitude of the recovery load, particularly in very soft clays (Small et al., 2015).

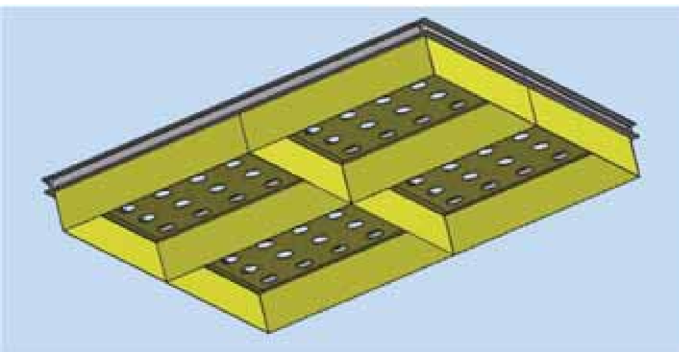

Figure 1. Schematic representation of a skirted subsea foundation

Much knowledge of uplift loads derives from work carried out on non-skirted footings and spudcans. Das (1990) incorporated an empirical time factor to match predictions and data for flat-based footings. Lehane et al. (2008) observed the opening of a gap beneath buried model footings being uplifted and identified this as accelerating the transition from undrained to drained behaviour. Fagundes et al. (2012) observed how perforations in model mudmats also accelerated this transition. Despite these careful studies, Bouwmeester et al. (2009) reported that the Das model gave a significant under prediction in uplift loads when applied to field trials on a seabed template. The guidelines produced by Naval Facilities Engineering Command, NAVFAC (2012), recommended calculating required uplift loads in terms of multiples of the in-service loads to which any given footing had been subjected.

A limited number of sources consider the deliberate uplift and breakout of skirted footings/suction caissons explicitly. Chen and Randolph (2007) observed benefits of cyclic extraction for suction caissons, attributed to radial stress redistribution. Samui et al. (2011) produced an empirical fit to an undeclared data set related to suction caissons. Gourvenec et al. (2009) performed a detailed study of centrifuge model footings in which preload was again identified as a factor, although in this study it was linked to accelerated extraction rate rather than a measure of required load.

1.1 Aim

From a review of the literature it is noted that no studies examine square-based footings of the sort identified earlier in this paper, and the role of internal skirts on the uplift process is also unclear. The aim of this work is to better bound the problem in relation to these structures. To meet this aim, a series of 1-g physical model tests in clay of square-based skirted footings with and without internal skirts was carried out using an Instron load frame at the University of Dundee.

2. Experimental Design and Procedure

2.1 Soil preparation

The soil type of most interest for this study was a very soft clay, in line with common conditions where such activities will occur. The chosen laboratory soil was a Speswhite kaolin, properties of which are in Table 1 based on the study by Robinson and Brown (2013). This was prepared by mixing with water at 100% moisture content in 25 kg batches to form a slurry.

Consolidation and testing occurred in a 650 mm diameter cylindrical steel container. A drainage layer of gravel and a filter layer of porous plastic and filter paper were first placed in the container. The clay slurry was then poured slowly into the container underwater to avoid air entrainment. Once 150 kg of slurry had been added, the mixture was left to settle overnight, before being gradually consolidated under increasing loads up to a target vertical confining pressure of 43 kPa. Loading was applied through a stiff, heavy lid and a hydraulic press, and was increased in steps over a period of three weeks until settlement increments became too small to measure. This process produced clay layers thicker than the target minimum requirement of 160 mm, based on ensuring structures did not experience base boundary effects. A hand vane test was performed at this stage to confirm suitable soil conditions.

Table 1. Selected properties of Speswhite kaolin

Property	Value
Plastic limit w_P (%)	32.5
Liquid limit w_L (%)	65.0
Specific gravity G_s	2.55
c_v (m^2/yr) (100 kPa stress increment)	23.5
Permeability k (m/s) (at σ' = 300 kPa)	1.2×10^{-9}

2.2 Model structures

Typical breadth of foundations for subsea structures would be 2–10 m. Skirt length could vary from very small ribs of 0.1 m to very long skirts of 2–3 m. Model structures were designed to be 80 mm square in plan, with skirt penetration of 5 mm, 20 mm and 80 mm into the soil. This was based on a bearing capacity limit analysis in which 80 mm was the largest size that would permit multiple footings to be tested in the same soil bed without interference between models/boundaries for the deepest foundations. A single internal skirt on each axis was provided where required. The foundations were fabricated from steel, roughened with shot blasting and allowed to rust in the model's water before testing to ensure a rough interface. This surface preparation produced interface properties consistent with the as-installed condition for full scale foundations. The top of the structure was provided with 20 M6 tapped holes that could be either open or closed, to enable control of the surface drainage condition. The fully open case corresponds to an open area of 8.8% of plan area. Figure 2 shows one of the model structures (pre-surface preparation) on its side, so internal skirts and top holes are visible.

Figure 2. A model structure from below, prior to roughening

2.3 Testing procedure

The prepared soil bed was transferred to the Instron load frame. The foundation was attached to the loading jaws with all drainage holes open. This was lowered until the structure was completely submerged below water but not touching the soil, at which point the load measurement was set to zero to eliminate structure buoyant weight from measurements. Sufficient water existed above the soil surface to ensure that even the foundations with the longest skirts could be fully extracted from the soil while remaining buoyant. The structure was then lowered at a rate of 1 mm/min until the soil contacted the flat base of the top plate, indicated by a sudden increase in required installation load. When tests required a preload, this was applied under load control at this stage using the Instron. Preload duration was calculated to ensure at least 90% of consolidation should have occurred within the soil, based on skirt depth D as an indicative drainage path length. This permitted any strength changes to manifest themselves although did not allow any additional bonding between soil and structure.

Two types of extraction tests were completed. The first type was a "drained" test in which the model was uplifted at 1 mm/min, the slowest possible rate, with holes open. The second type was "undrained", in which the holes were closed by M6 bolts, then uplift carried out at 30 mm/min. Whether these descriptions of drainage regime are appropriate is discussed later.

After all the testing had been completed for a given soil bed, three T-bar tests were carried out using a 10 mm diameter model T-bar, followed by hand vane and moisture content measurements. A design undrained shear strength (s_u) profile was assigned to each individual model to account for variations between test beds. s_u values varied from 0.9 - 2.9 kPa at surface, increasing at gradients of 0.8 – 2.2 kPa per 100 mm depth.

2.4 Test series

Table 2 shows the combinations of embedment ratio (defined as footing breadth B/penetration depth D), drainage condition ("drained" (D) or "undrained" (U) as defined above, or "partially drained" (P) which was uplifted at undrained rate but with four corner holes left open), preload, and number of internal skirts on each side that were tested.

Table 2. Programme of tests undertaken

Test	B/D	Drainage	Preload	Int. skirts
1-20-U-0	4	U	0	1
1-80-U-0	1	U	0	1
2-20-D-0	4	D	0	1
2-80-D-0	1	D	0	1
3-05-D-22	16	D	$1.s_u$	1
3-05-U-22	16	U	$1.s_u$	1
3-05-P-22	16	P	$1.s_u$	1
4-20-D-48	4	D	$3.s_u$	1
4-20nis-D-48	4	D	$3.s_u$	0
4-20nis-U-48	4	U	$3.s_u$	0

3. Uplift Prediction

The test series provides measurement of net uplift forces for the tests considered. These loads may also be estimated analytically. Das (1991) broke down the components of load as: uplift load = buoyant weight of structure + side adhesion + base suctions. This provides a useful starting point for breaking down resistance components. Here, the buoyant structure weight was subtracted, but it was noted that in a number of tests the structure retained a soil plug. Figure 3a shows an extracted foundation that has retained a plug, compared to Figure 3b which has not. It is therefore reasoned that if the foundation is to extract as plugged then the mechanism may differ. Figure 4 illustrates load components considered to be acting for (a) unplugged and (b) plugged extraction.

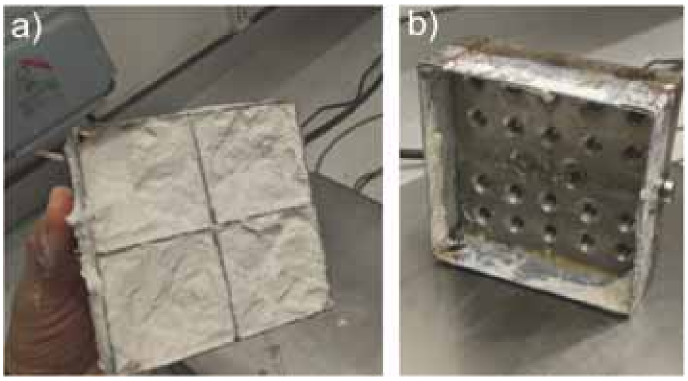

Figure 3. Foundations post-extraction a) plugged and b) unplugged

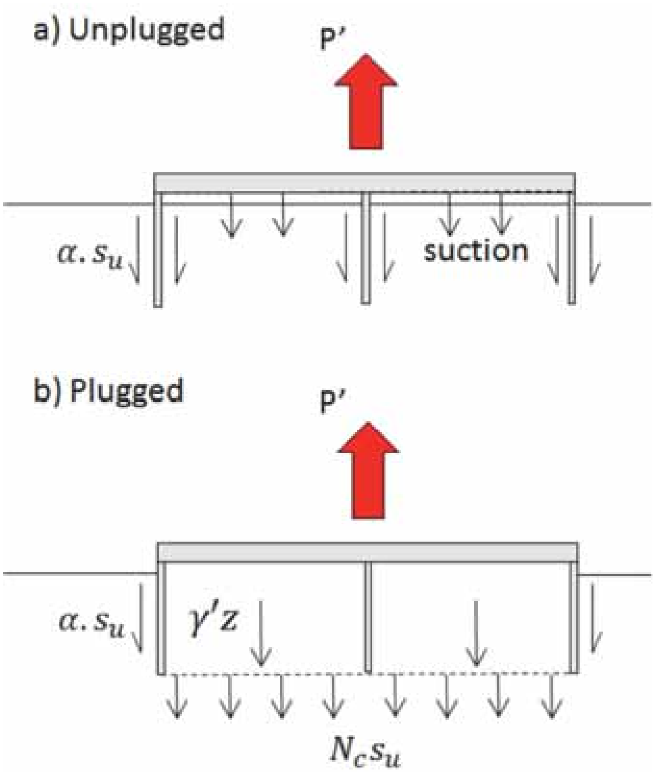

Figure 4. Components of recovery load when extraction is a) unplugged and b) plugged

In Figure 4, recovery load (minus submerged structure weight) is denoted as P′, shear on vertical surfaces which occurs on external skirts and (when unplugged) internal skirts is $\alpha.s_u$ where α is an adhesion factor and s_u is soil undrained shear strength, γ' is soil submerged unit weight, z is depth below seabed and N_c is a bearing capacity factor for a reverse end-bearing failure. For a square structure of breadth B, embedment D and with n internal skirts per side (i.e. the model in Figure 2 has n = 1 and in Figure 3b has n = 0), formulating the load components shown in Figure 4 gives, for unplugged extraction:

$$P' = \alpha.s_u.(4BD) + \alpha.s_u.(4BD + 4BD \times n) + \text{suctions} \quad (1)$$

And for plugged extraction:

$$P' = \alpha.s_u.(4BD) + (N_c s_u - \gamma' D) \times B^2 + \gamma'.DB^2 \quad (2)$$

which simplifies to

$$P' = \alpha.s_u.(4BD) + N_c s_u B^2 \quad (3)$$

As undrained shear strength varies with depth, values were taken at z = D/2 for the side wall adhesion and at z = D + B/4 for the end-bearing term, the latter being in line with Martin (1994) on compressive bearing capacity. It was therefore possible to predict peak recovery load for each model based on these individual load components, if the unknown "suctions" in Equation 1 are set to zero to account for the open holes. Adhesion factor α was back-calculated from installation data, where after an initial transient phase it tended to a steady state value between 0.75 and 1. For consistency, α and taken as 1 for analysis presented below.

Prediction of recovery load could also be carried out based on the preload-dependent method of NAVFAC (2012) which for a preload of Q gives, for $B/D \geq 4$:

$$P' = 2(Q/2) + \text{attached soil weight} \quad (4)$$

And for $B/D < 4$:

$$P' = 2Q(1 - \exp(-2.75D/B) + \text{attached soil weight} \quad (5)$$

In order to assess the validity of each of these methods, calculations were performed and compared to the peak loads measured in the test series. The results of these are plotted in Figure 5.

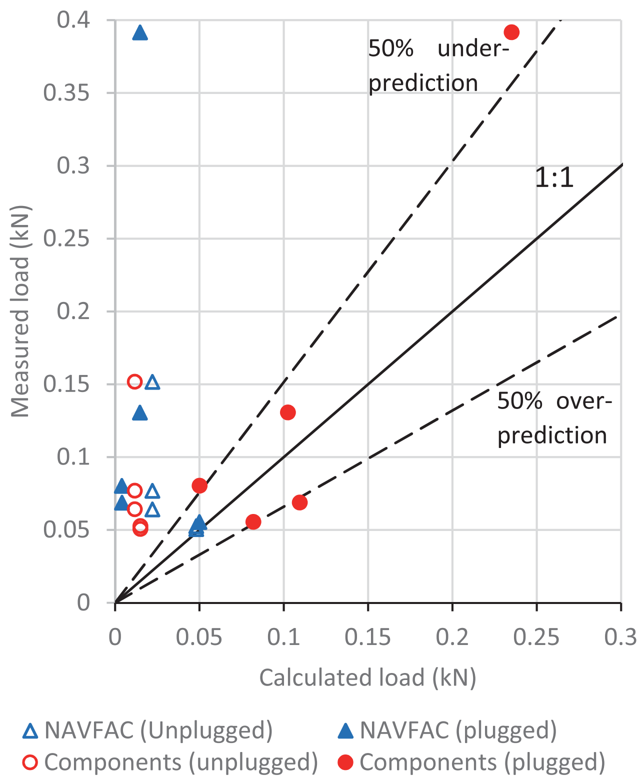

Figure 5. Measured loads vs. loads predicted using individual components (Eq. 1, 3) and preloads after NAVFAC (Eq. 4, 5)

Figure 5 shows one value plotted against the other, with a perfect prediction being along the solid 1:1 line. To give a guide to tolerance, dashed lines representing a 50% over- or under-prediction are also added. Distinction is drawn between tests that occurred plugged or unplugged by using closed or open symbols accordingly.

Considering first the NAVFAC predictions based on preload (Equations 4 and 5) it is notable that the only reasonable estimates from the preload method come from the tests that experienced the greatest preload (the filled triangles on the 1:1 line in Figure 5). These estimates are excellent and superior to the values obtained using load components. However, in general, assessing the load components gives a better chance of reproducing the load measured, which is to be expected as it directly considers the mechanics of the problem.

3.1 Role of preloading

It may be possible that the preload method is able to show a good empirical match as a result of chance. Preload for these tests was selected in order to obtain an approximate working load equal to 50% of capacity i.e. a design factor of safety of 2. This meant that the value of preload was calculated as 3.su (equivalent to a bearing capacity factor of 6). Existing foundations designed to an operating factor of safety of 2 will therefore have been subjected to a similar preload. Thus when using the preload term in the expressions, they become a function of $3.s_u$ rather than preload, which is now a similar form to the component equations which are also functions of s_u. If the net coefficient of s_u, when all the components are added up, comes close to 3 then the two methods will return a similar result. Figure 6 shows the load-displacement data from two tests carried out with identical properties except that one had been subject to a preload prior to recovery (tests 2-20-D-0 and 4-20-D-48 in Table 2). Figure 6 shows that there was little significant difference between the drained load-displacement results from a preloaded and non-preloaded foundation. Therefore, it is not certain that the use of preload is representative of any actual response but rather a coincidental feature based on the common practices in subsea structure design.

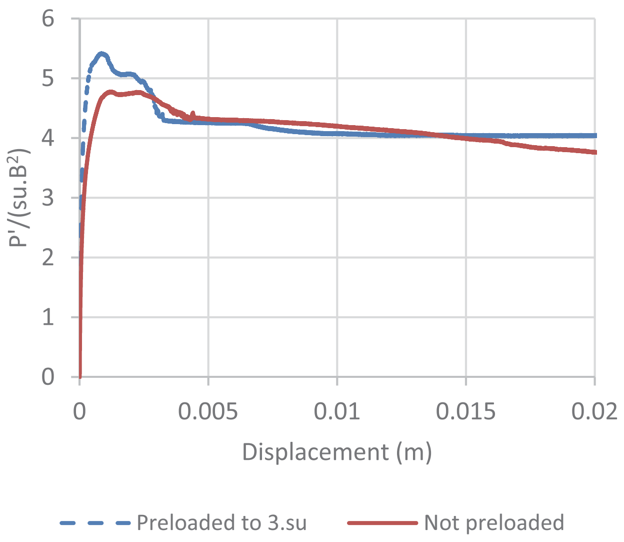

Figure 6. Normalised drained uplift force-displacement behaviour of differently preloaded foundations

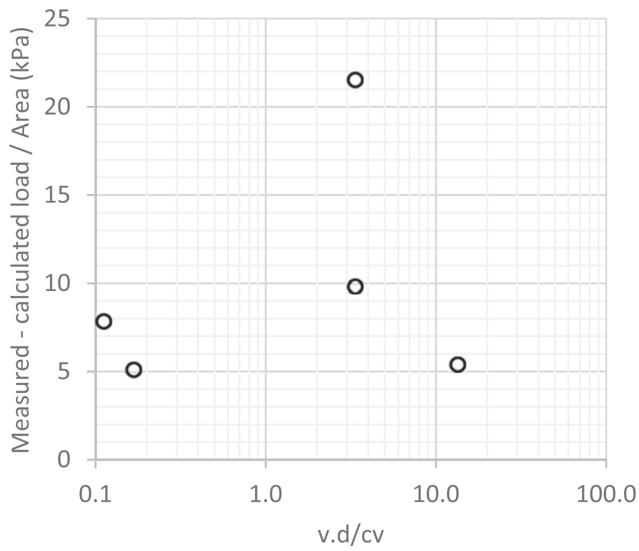

Figure 7. Back calculated suction pressures as a function of drainage parameters for unplugged tests

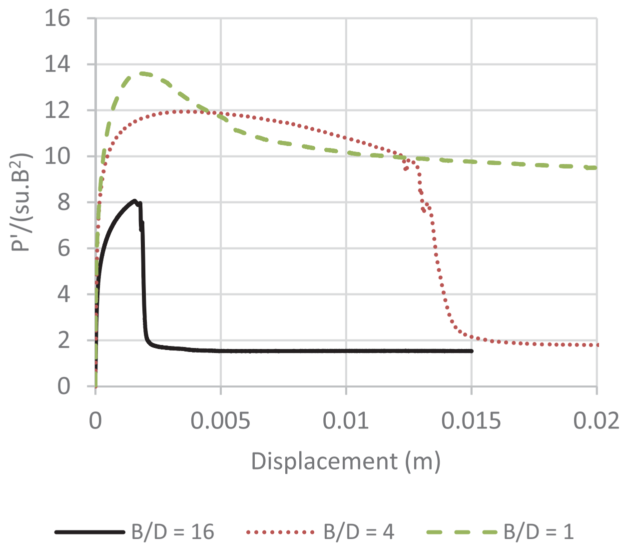

Figure 8. Normalised undrained uplift force-displacement behaviour of different skirt lengths

3.2 Rate effects

The components calculated for the plugged uplift are superior predictions to those for the unplugged uplift, which are less reliable. The difference is the unknown amount of suction that was not calculated, but clearly controls response even when extraction is slow and intended to be "drained". Uplift was limited to a slowest possible rate of 1 mm/min, available using this equipment configuration. Back calculated values for this apparent suction have been determined in Figure 7 by taking the difference between measured load and known load components, and dividing by base area to determine a pressure (note that all base areas are the same in this study). These pressures are plotted against normalized velocity V = $v.d/c_v$ which is expected to govern whether loading is drained, undrained or partially drained. For these tests, values of d have been determined based on skirt length D for the undrained tests and half the hole pitch in the top surface for the drained tests. Almost all data lies in the range 5-10 kPa, with one outlier, from the test 3-05-U-22, the very shortest skirt, which can be seen in Figure 8 to be tending towards the load of the deeper foundations before the suction breaks. The large discrepancy shown in Figure 7 is therefore due to the three foundations (of identical base area) all pulling a similar amount of suction (Figure 8) but this particular test predicting a very small resistance due to the short skirts. Figure 5 suggests that the suction component is the overriding factor in unplugged extraction; Figure 8 suggests that this is a function only of the plan area, as long as suction does not break.

In testing flat (unskirted) footings on clay soils but beneath a sand surcharge, Lehane et al. (2008) defined a bearing term N_u, which would be multiplied by $s_u.B^2$ to obtain an estimate of suction force. Lehane et al. showed values of N_u being negligible at a normalized velocity of 10 and rising by 1 per log increase in V thereafter. This was based on a drainage path length equalling footing width (d = B) in the absence of skirts or holes. N_u has been back-calculated from these test in order to identify any consistency in suctions. This has been achieved by evaluating the term "suctions" in Equation 1 for each test and normalizing by $s_u.B^2$ (where s_u was evaluated at footing base level), and plotted as a function of $v.d/c_v$ in Figure 9.

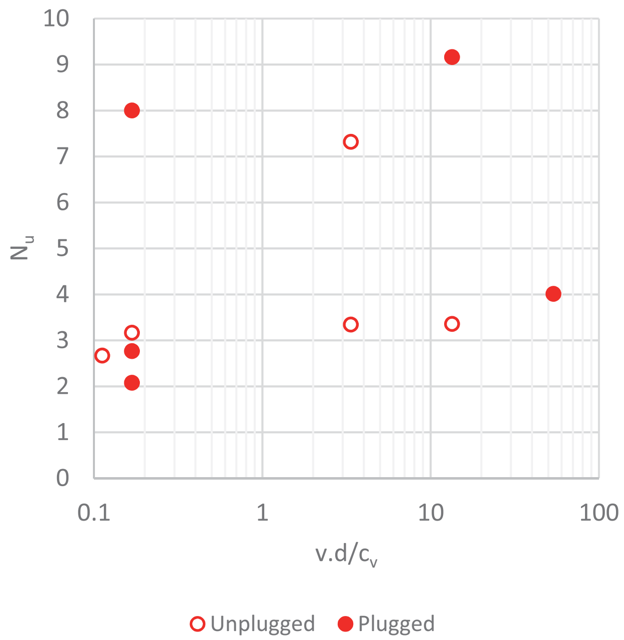

Figure 9. Bearing factor for suctions, N_u

Figure 9 shows that across the range of normalized velocities tested, N_u rises more gently than the 1 per log cycle shown in Lehane et al., but also is prone to being significantly higher. The three tests showing very high apparent suctions (1-20-U-0, 2-80-D-0 and 3-05-U-22 in Table 2) have little in common, indicating that prediction of suctions is more complex than can be handled by a bearing factor in this case. It is also seen that the normalized velocity of 10 at which the flat plate was shown to start experiencing suctions, is by no means the lower limit for suction in the case of the skirted footing.

4. Role of Plugging

4.1 Determining plugged behaviour

Due to the differences identified above, it becomes useful to distinguish at the earliest possible stage whether or not a foundation is likely to uplift in an unplugged or a plugged manner. For quick differentiation between the cases, Figure 10 plots occurrences of unplugged (open symbols) and plugged (filled symbols) based on their drainage conditions and their internal aspect ratio, where this is defined as the separation between skirts divided by skirt depth. This has been chosen as it is the internal sections that hold the plug and will therefore be the governing aspect ratio, rather than any more global properties. Table 3 shows how this relates to the tests identified in Table 2. Figure 10 shows a clear distinction between the two modes as well as clear increases in uplift loads when soil plugs are retained.

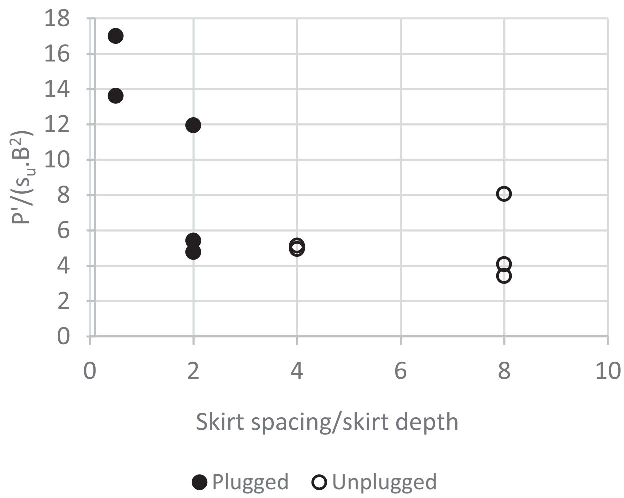

Figure 10. Relationship between skirt spacing and plugging behaviour in this study

Table 3. Results of tests undertaken

Test	Plugging	Spacing/D	$P'/(s_u^2.B)$
1-20-U-0	Plugged	2	11.9
1-80-U-0	Plugged	0.5	13.6
2-20-D-0	Plugged	2	4.8
2-80-D-0	Plugged	0.5	17.0
3-05-D-22	Unplugged	8	3.4
3-05-U-22	Unplugged	8	8.1
3-05-P-22	Unplugged	8	4.1
4-20-D-48	Plugged	2	5.4
4-20nis-D-48	Unplugged	4	4.9
4-20nis-U-48	Unplugged	4	5.1

For this study, plugging depended on the ratio of inter-skirt spacing to skirt depth. Ratios of $B/D > 4$ produced unplugged behaviour. Ratios of $B/D < 2$ or less always plugged. The process will of course be controlled by internal shear, suction pressure and the reverse end-bearing capacity, so this cut off ratio may differ in other soil-structure systems at larger scale, with shear strength gradient being the likely most important factor because of the relative role of end-bearing shear beneath the skirt tip compared to interface shear above the skirt tip.

4.2 Importance of plugging

The previously detailed analysis is only partially satisfactory because (i) suction pressures are still not known and (ii) plugging depends on soil strength/drainage parameters and geometric estimates present only a “rule of thumb” type guide. However, it is acknowledged that the transition from unplugged to plugged occurs when the reverse end-bearing failure begins, and that this therefore represents an upper bound on the amount of suction that can be generated. If the design value of uplift load sought is preferably an upper bound (it being preferable to cater for too much force than too little), then calculating all uplift loads as if they were plugged (even if they are not) should provide an upper limit. To investigate the veracity of this, the uplift calculation was repeated, but this time treating all cases as if plugged (Equation 3). These values are plotted in Figure 11.

Figure 11 shows a much better match between the predicted uplift loads and those that were measured compared to those achieved previously when considering the exact loads occurring in each case. As expected, the unplugged cases are generally now overestimated because the suctions were not quite high enough to produce the end-bearing failure considered. However, this is an improvement on the previous estimate and an upper bound may be permissible for planning purposes.

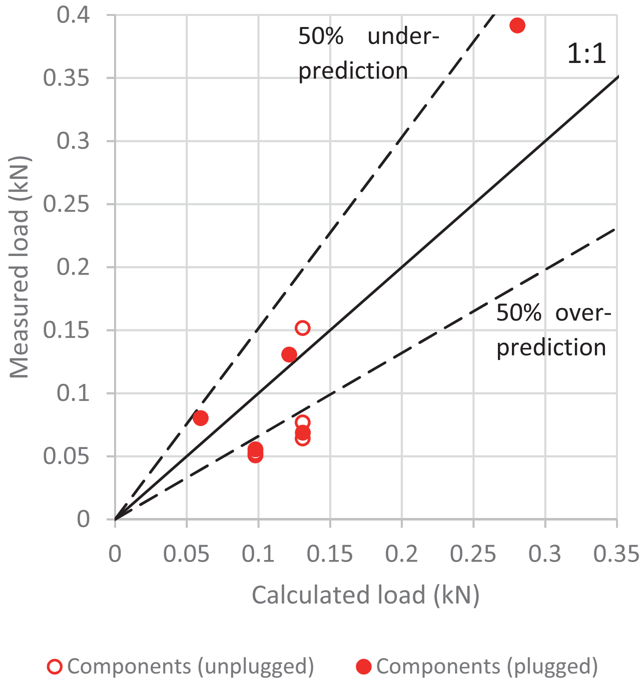

Figure 11. Measured loads compared to calculated loads, when calculation assumes plugged behaviour at all times (Equation 3)

4.3 Variability of parameters

The method shown above is intended as an upper bound and appears to behave as such for data measured. Parameters for base bearing capacity N_c, adhesion α and undrained shear strength at side and below the base, s_u may vary. N_c is relatively well established and alternatives to the Salgado et al. (2004) formulation produced values within 5%, indicating that this is a stable parameter. Adhesion α, as stated above, could have a value between 0.75 and 1, i.e. up to 25% less than the value of 1 used for calculation. This affects only the resistance acting on the skirt sides, and the relative influence depends on the relative area of sides (4.B.D) to base area (B^2) and the gradient of shear strength increase. Undrained shear strength is always reliant on good-quality measurements, both in the model tests here and in practice. The predictions vary linearly with this parameter (Equation 3) so any under measurement in s_u will result in a corresponding under prediction in recovery load.

5. Conclusions

A test method was created for evaluating uplift forces encountered during removal of skirted shallow footings from clay seabeds. The small scale modelling should produce results that scale up to larger values if the soil is treated as a plastic continuum (as is common in numerical modelling) and appropriate parameters are non-dimensionalized.

Uplift values calculated using empirical estimates based on foundation working load were found to be inaccurate. It is suggested that these work because of uniformity in the design process, which meant that the working load related to soil properties such that it was acting as an indirect measure of soil strength rather than as a parameter in its own right. In directly comparable tests in this study, preload had a minor effect on response that was indistinguishable when the above method was employed.

Uplift tests showed that the most important factor in calculating uplift load precisely was plugging. This could occur even when uplift was slow and with drainage permitted, and shown to most closely correlate with the ratio of inter-skirt spacing to skirt depth. Ratios of 4 and more produced unplugged behaviour. Ratios of 2 or less always plugged, for the conditions tested. When extracting plugged, loads were predictable by consideration of the individual components of load. However, when uplifting without a soil plug then the unknown amount of suction pressure dominated the uplift load. This suction proved too unpredictable to model using a bearing factor such as N_u. Consideration of the limiting value of this suction showed that if all cases were treated as plugged then prediction became far more reliable albeit providing an upper bound on the expected uplift force that might be achieved.

The test series demonstrated the value of physical modelling for decommissioning engineering and that small scale physical modelling could be adopted as a feasible method of assessing recovery loads. The study also demonstrated the importance of obtaining and maintaining foundation installation records as an input into future decommissioning studies.

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References

Bouwmeester D, Peuchen J, Van der Wal T, Sarata B, Willemse CA, Van Baars S and Peelen R. (2009) Prediction of breakout forces for deepwater seafloor objects *Proc. OTC 2009*, paper no. OC 19925

Chen W and Randolph MF. (2007) Uplift capacity of suction caissons under sustained and cyclic loading in soft clay. *ASCE J. Geotech. & Geoenviron. Eng.* 133(11):1352-363

Craig WH and Chua K. (1990) Extraction forces for offshore foundations under undrained loading *ASCE J. Geotech. Eng.* 116(5): 868-884

Das BM. (1991) Bottom breakout of objects resting on soft clay sediments *International Journal of Offshore and Polar Engineering* 1(3)

Fagundes D de F, Almeida MCF, Almeida MSS and Rammah KI. (2012) Physical modelling of offshore structures founded on sea bed *Proc. OMAE 2012*, Paper no. OMAE2012-83265

Gourvenec S, Acosta-Martinez HE and Randolph MF. (2009) Experimental study of uplift resistance of shallow skirted foundations in clay under transient and sustained concentric loading *Géotechnique* 59(6): 525-537

Lehane BM, Gaudin C, Richards DJ and Rattley MJ. (2008) Rate effects on the vertical uplift capacity of footings founded in clay. *Géotechnique* 58(1): 13-21

Lehane BM, O'Loughlin CD, Gaudin C and Randolph MF. (2009) Rate effects on penetrometer resistance in kaolin. *Géotechnique* 59(1): 41-52

Li X, Tian Y, Gaudin C and Cassidy MJ. (2015) Comparative study of the compression and uplift of shallow foundations *Computers and Geotechnics* 69: 38-45

Martin CM. (1994) Physical and numerical modelling of offshore foundations under combined loads DPhil thesis, University of Oxford, UK.

NAVFAC (2012) *Handbook for marine geotechnical engineering* Naval Facilities Engineering Command Report no. SP-2209-OCN

Purwana OA, Leung CF, Chow YK and Foo KS. (2005) Influence of base suction on extraction of jack-up spudcans *Géotechnique* 55(10): 741-753

Rapoport V and Young AG. (1985) Uplift capacity of shallow offshore foundations. *Proc. ASCE Convention, Session: Uplift Behavior Anchor Foundations in Soil*, Detroit, Michigan, 1985, pp 73-85

Robinson S and Brown MJ. (2013) Rate effects at varying strain levels in fine grained soils. *Proc. 18th Int. Conf. on Soil Mechanics and Geotechnical Engineering*, Paris 2013. pg. 263 – 266

Salgado R, Lyamin AV, Sloan SW and Yu HS. (2004) Two and three dimensional bearing capacity of foundations in clay *Géotechnique* 54(5): 297-306

Samui P, Das S and Dookie K. (2011) Uplift capacity of suction caisson in clay using multivariate adaptive regression spline *Ocean Engineering* 38: 2123-2127

Small A, Cook G, Egborge R and Ejidike A. (2015) Geotechnical aspects of North Sea decommissioning *Proc. ISOPE 2015*, pg. 1308-1314